SPECIFICATIONS	COMMENTARY
5.1 SCOPE	C5.1
5.2 DEFINITIONS	C5.2
5.3 NOTATION	C5.3
5.4 MATERIAL PROPERTIES	C5.4
5.4.1 General	C5.4.1
5.4.2 Normal and Structural Low-Density Concrete	C5.4.2
5.4.2.1 COMPRESSIVE STRENGTH The following shall be added to paragraph 3. <u>The specified compressive strength for reinforced concrete shall not be less than 3.60 ksi (25Mpa).</u>	C5.4.2.1
5.4.2.2 COEFFICIENT OF THERMAL EXPANSION	C5.4.2.2
5.4.2.3 SHRINKAGE AND CREEP	C5.4.2.3
5.4.2.3.1 General	C5.4.2.3.1
5.4.2.3.2 Creep	C5.4.2.3.2
5.4.2.3.3 Shrinkage	C5.4.2.3.3
5.4.2.4 MODULUS OF ELASTICIY	C5.4.2.4
5.4.2.5 POISSON'S RATIO	C5.4.2.5
5.4.2.6 MODULUS OF RUPTURE	C5.4.2.6
5.4.2.7 TENSILE STRENGTH	C5.4.2.7
5.4.3 Reinforcing Steel	C5.4.3
5.4.3.1 GENERAL	C5.4.3.1
5.4.3.2 MODULUS OF ELASTICITY	C5.4.3.2
5.4.3.3 SPECIAL APPLICATIONS	C5.4.3.3
5.4.4 Prestressing Steel	C5.4.4
5.4.4.1 GENERAL	C5.4.4.1
5.4.4.2 MODULUS OF ELASTICITY	C5.4.4.2

SPECIFICATIONS	COMMENTARY
5.4.5 Posttensioning Anchorages and Couplers	C5.4.5
5.4.6 Ducts	C5.4.6
5.4.6.1 GENERAL	C5.4.6.1
5.4.6.2 SIZE OF DUCTS	C5.4.6.2
5.4.6.3 DUCTS AT DEVIATION SADDLES	C5.4.6.3
5.5 LIMIT STATES	C5.5
5.5.1 General	C5.5.1
5.5.2 Service Limit State	C5.5.2
5.5.3 Fatigue Limit State	C5.5.3
5.5.3.1 GENERAL	C5.5.3.1
5.5.3.2 REINFORCING BARS	C5.5.3.2
5.5.3.3 PRESTRESSING TENDONS	C5.5.3.3
5.5.3.4 WELDED OR MECHANICAL SPLICES OF REINFORCEMENT	C5.5.3.4

SPECIFICATIONS

5.5.4 Strength Limit State

5.5.4.1 GENERAL

5.5.4.2 RESISTANCE FACTORS

5.5.4.2.1 Conventional Construction

Modify as follows:

- For flexure and tension of reinforced concrete……………………..0.90
- For flexure and tension of cast-in-place prestressed concrete…..0.95
- For flexure and tension of precast prestressed concrete………..1.00

rewording, approval pending

5.5.4.2.2 Segmental Construction

5.5.4.2.3 Special Requirements For Seismic Zones 3 and 4

5.5.4.3 STABILITY

COMMENTARY

C5.5.4

C5.5.4.1

C5.5.4.2

C5.5.4.2.1

C5.5.4.2.2

C5.5.4.2.3

C5.5.4.3

SPECIFICATIONS	COMMENTARY
5.5.5. Extreme Event Limit State	C5.5.5
Modify as follows: The structure as a whole and its components shall be proportioned to resist collapse due to extreme events, specified in Table 3.4.1-1, as may be appropriate to its site and use. <u>Resistance factors shall be 1.0.</u>	C5.6
5.6 DESIGN CONSIDERATIONS	
5.6.1 General	C5.6.1
5.6.2 Effects of Imposed Deformation	C5.6.2
5.6.3 Strut-and-Tie Model	C5.6.3
5.6.3.1 GENERAL	C5.6.3.1
5.6.3.2 STRUCTURAL MODELING	C5.6.3.2
5.6.3.3 PROPORTIONING OF COMPRESSIVE STRUTS	C5.6.3.3
5.6.3.3.1 Strength of Unreinforced Strut	C5.6.3.3.1
5.6.3.3.2 Effective Cross-Sectional Area of Strut	C5.6.3.3.2
5.6.3.3.3 Limiting Compressive Stress in Strut	C5.6.3.3.3
5.6.3.3.4 Reinforced Strut	C5.6.3.3.4
5.6.3.4 PROPORTIONING OF TENSION TIES	C5.6.3.4
5.6.3.4.1 Strength of Tie	C5.6.3.4.1
5.6.3.4.2 Anchorage of Tie	C5.6.3.4.2
	C5.6.3.6

SPECIFICATIONS

5.7 DESIGN FOR FLEXURAL AND AXIAL FORCE EFFECTS

5.7.1 Assumptions for Service and Fatigue Limit States

5.7.2 Assumptions for Strength and Extreme Event Limit States

5.7.2.1 GENERAL

Add bulleted items to the end of the list in Article 5.7.2.1 as follows:

- Balanced strain conditions exist at a cross section when tension reinforcement reaches the strain corresponding to its specified yield strength f_y just as the concrete in compression reaches its assumed ultimate strain of 0.003.
- Sections are compression-controlled when the net tensile strain in the extreme tension steel is equal to or less than the compression-controlled strain limit at the time the concrete in compression reaches its assumed strain limit of 0.003. The compression-controlled strain limit is the net tensile strain in the reinforcement at balanced strain conditions. For Grade 60 reinforcement, and for all prestressed reinforcement, the compression-controlled strain limit may be set equal to 0.002.
- Sections are tension-controlled when the net tensile strain in the extreme tension steel is equal to or greater than 0.005 just as the concrete in compression reaches its assumed strain limit of 0.003. Sections with net tensile strain in the extreme tension steel between the compression-controlled strain limit and 0.005 constitute a transition region between compression-controlled and tension-controlled sections.

The use of compression reinforcement in conjunction with additional tension reinforcement is permitted to increase the strength of flexural members.

COMMENTARY

C5.7

C5.7.1

C5.7.2

C5.7.2.1

Add commentary to Article C5.7.2.1 to accompany the CA bulleted items as follows:

The nominal flexural strength of a member is reached when the strain in the extreme compression fiber reaches the assumed strain limit of 0.003. The net tensile strain ε_t is the tensile strain in the extreme tension steel at nominal strength, exclusive of strains due to prestress, creep, shrinkage, and temperature. The net tensile strain in the extreme tension steel is determined from a linear strain distribution at nominal strength, as shown in Figure C5.7.2.1-1, using similar triangles.

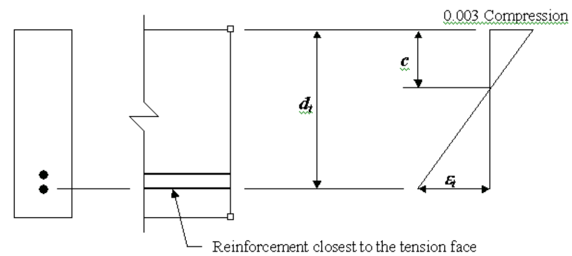

Figure C5.7.2.1-1 – Strain distribution and net tensile strain

When the net tensile strain in the extreme tension steel is sufficiently large (equal to or greater than 0.005), the section is defined as tension-controlled where ample warning of failure with excessive deflection and cracking may be expected. When the net tensile strain in the extreme tension steel is small (less than or equal to the compression-controlled strain limit), a brittle failure condition may be expected, with little warning of impending failure. Flexural members are usually tension-controlled, while compression members are usually compression-controlled. Some sections, such as those with small axial load and large bending moment, will have net tensile strain in the extreme tension steel between the above limits. These sections are in a transition region between compression- and tension-controlled sections. Article 5.5.4.2.1 specifies the appropriate resistance factors for tension-controlled and compression-controlled sections, and for intermediate cases in the transition region.

SPECIFICATIONS

Before the development of these provisions, the limiting tensile strain for flexural members was not stated, but was implicit in the maximum reinforcement limit that was given as $c/d_e \leq 0.42$, which corresponded to a net tensile strain at the centroid of the tension reinforcement of 0.00414. The net tensile strain limit of 0.005 for tension-controlled sections was chosen to be a single value that applies to all types of steel (prestressed and nonprestressed) permitted by this specification.

Unless unusual amounts of ductility are required, the 0.005 limit will provide ductile behavior for most designs. One condition where greater ductile

COMMENTARY

behavior is required is in design for redistribution of moments in continuous members and frames. Article 5.7.3.5 permits redistribution of negative moments. Since moment redistribution is dependent on adequate ductility in hinge regions, moment redistribution is limited to sections that have a net tensile strain of at least 0.0075.

For beams with compression reinforcement, or T-beams, the effects of compression reinforcement and flanges are automatically accounted for in the computation of net tensile strain ε_t.

5.7.2.2 RECTANGLULAR STRESS DISTRIBTION

C5.7.2.2

SPECIFICATIONS

5.7.3 Flexural Members

5.7.3.1 STRESS IN PRESTRESSING STEEL AT NOMINLA FLEXURAL RESISTANCE

5.7.3.1.1 Components with Bonded Tendons

Modify Eqn 3 as shown:

$$c = \frac{A_{ps} f_{pu} + A_s f_y - A'_s f'_y - 0.85 \cancel{\beta_1} f'_c (b - b_w) h_f}{0.85 f'_c \beta_1 b_w + k A_{ps} \dfrac{f_{pu}}{d_p}}$$

Approval pending!

COMMENTARY

C5.7.3

C5.7.3.1

C5.7.3.1.1

Add the following as the last paragraph:

Studies have been done showing that the Unified Concrete Methodology in AASHTO leads to flexural capacities approximately 15% less than those evaluated using the AASHTO Std. Specs or ACI. The cause seems to be the assumption that flanged behavior begins when the neutral axis, rather than the compression block, falls outside the tension flange. The difference is independent of the increase from HS20 to HL93. On AASHTO SCOBS'05 ballot.

SPECIFICATIONS

5.7.3.1.2 Components with Unbonded Tendons

Modify Eqn. 3 as shown:

$$c = \frac{A_{ps} f_{ps} + A_s f_y - A'_s \cancel{f_y} - 0.85 \beta_1 f'_c (b - b_w) h_f}{0.85 f'_c \beta_1 b_w}$$

Approval pending!

5.7.3.2 FLEXURAL RESISTANCE

5.7.3.2.1 Factored Flexural Resistance

For flanged sections subjected to flexure about one axis and for biaxial flexure with axial load as specified in Article 5.7.4.5, where the approximate stress distribution specified in Article 5.7.2.2 is used ~~and the tendons are bonded~~ and where the compression flange depth is less than ~~c~~ $a=\beta_1 c$, as determined in accordance with Equations 5.7.3.1.1-3,

$$M_n = A_{ps} f_{ps}\left(d_p - \frac{a}{2}\right) + A_s f_y\left(d_s - \frac{a}{2}\right) - A'_s f'_y\left(d'_s - \frac{a}{2}\right) + 0.85 f'_c (b - b_w)\cancel{\beta_1} h_f\left(\frac{a}{2} - \frac{h_f}{2}\right)$$

Approval pending!

5.7.3.2.2 Flanged Sections

In Equation 1, A_s'=0.

5.7.3.2.3 Rectangular Sections

5.7.3.2.4 Other Cross-Sections

COMMENTARY

C5.7.3.1.2

C5.7.3.2

C5.7.3.2.1

5.7.3.1.1-4, 5.7.3.1.2-3 or 5.7.3.1.2-4, the nominal flexural resistance may be taken as:

C5.7.3.2.2 In previous editions and interims of the LRFD Specifications, the factor β_1 was applied to the flange overhang term of Equations 1, 5.7.3.1.1-3 and 5.7.3.1.2-3. This was not consistent with the original derivation of the equivalent rectangular stress block as it applies to flanged sections (Mattock, Kriz and Hognestad 1961). For the current LRFD Specification, the β_1 factor has been removed from the flange overhang term of these equations. See also Seguirant (2002), Girgis, Sun and Tadros (2002), Naaman (2002), Weigel, Seguirant, Brice and Khaleghi (2003), Baran, Schultz and French (2004), and Seguirant, Brice and Khaleghi (2004).

Approval pending!

C5.7.3.2.3

C5.7.3.2.4

SPECIFICATIONS

5.7.3.3 LIMITS FOR REINFORCEMENT

5.7.3.3.1 Maximum Reinforcement

Remove all of Article 5.7.3.3.1 and replace with [on '05 ballot]

Approval pending!

COMMENTARY

C5.7.3.3

C5.7.3.3.1

Remove all of Article C5.7.3.3.1 and replace with:

In editions of and interims to the LRFD specifications prior to 2005, Article 5.7.3.3.1 limited the tension reinforcement quantity to a maximum amount such that the ratio c/d_e did not exceed 0.42. Sections with $c/d_e > 0.42$ were considered over-reinforced. Over-reinforced nonprestressed members were not allowed, whereas prestressed and partially prestressed members with PPR greater than 50 percent were if "it is shown by analysis and experimentation that sufficient ductility of the structure can be achieved." No guidance was given for what "sufficient ductility" should be, and it was not clear what value of ϕ should be used for such over-reinforced members.

The current provisions of LRFD eliminate this limit and unify the design of prestressed and nonprestressed tension- and compression-controlled members. The background and basis for these provisions are given in Mast (1992). Below a net tensile strain in the extreme tension steel of 0.005, as the tension reinforcement quantity increases, the factored resistance of prestressed and non-prestressed sections is reduced in accordance with Article 5.5.4.2.1. This reduction compensates for decreasing ductility with increasing over-strength. Only the addition of compression reinforcement in conjunction with additional tension reinforcement can result in an increase in the factored flexural resistance of the section.

5.7.3.3.2 Minimum Reinforcement

C5.7.3.3.2

SPECIFICATIONS

5.7.3.4 CONTROL OF CRACKING BY DISTRIBUTION OF REINFORCEMENT

5.7.3.4 Control of Cracking By Distribution of Reinforcement

The provisions specified herein shall apply to the reinforcement of all concrete components, except that of deck slabs designed in accordance with Article 9.7.2, in which tension in the cross-section exceeds 80 percent of the modulus of rupture, specified in Article 5.4.2.6, at applicable service limit state load combination specified in Table .

~~Components shall be so proportioned that the tensile stress in the mild steel reinforcement at the service limit state does not exceed f_{sa}, determined as:~~The spacing s of mild steel reinforcement in the layer closest to the tension fase shall satisfy the following:

$$\cancel{f_{sa} = \frac{Z}{(d_c A)^{\frac{1}{3}}} \le 0.6 f_y} \qquad s \le \frac{700\gamma_e}{\beta_s f_s} - 2d_c \qquad ($$

where

γ_e = exposure factor, 1.00 for Class 1, 0.75 for Class 2

d_c = thickness of concrete cover measured from extreme tension fiber to center of the flexural reinforcment located closest thereto

f_s = tensile stress in steel reinforcement at the service limit state (KSI)

$$\beta_s = 1 + \frac{d_c}{0.7(h - d_c)} \qquad (5.7.3.4\text{-}2)$$

~~Except as specified below for cast-in-place reinforced concrete box culverts, the quantity Z in Eq. 1 shall not exceed 170 kips/in. for members in moderate exposure conditions, 130 kips/in. for members in severe exposure conditions, and 100 kips/in. for buried structures. The quantity Z shall not exceed 130 for the transverse design of segmental concrete box girders for any loads applied prior to the attainment of the full nominal concrete strength. to the attainment of the full nominal concrete strength.~~

~~Except as specified…cast-in-place reinforced concrete box culverts, the quantity Z in Eq. 1 shall….~~

~~Bonded prestressing steel may be included in the calculation of A, in which case the increase in stress in the bonded prestressing steel beyond the decompression state calculated on the basis of a cracked section or strain compatibility analysis shall satisfy the value of f_{sa} determined from Eq. 1~~.

The effects of bonded prestressing steel may be considered, in which case the value of f_s used in Eq. 1, for the bonded prestressing steel, shall be the stress that develops beyond the decompression state calculated on the basis of a cracked section or strain compatibility analysis.

COMMENTARY

C5.7.3.4

excerpts from '05 Interims

Class 1 exposure condition applies when cracks can be tolerated due to reduced concerns of appearance and/or corrosion. Class 2 exposure condition applies to transverse design of segmental concrete box girders for any loads applied prior to attaining full nominal concrete strength and when there is increased concern of appearance and/or corrosion.

In the computation of d_c, the actual concrete cover thickness is to be used.

When computing the actual stress in the steel reinforcement, axial tension effects shall be considered, while axial compression effects may be considered.

The minimum and maximum spacing of reinforcement shall also comply with the provisions of Articles 5.10.3.1 and 5.10.3.2, respectively.

Eq. 1 is expected to provide a distribution of reinforcement that should control flexural cracking. The equation is based on a physical crack model (Frosch 2001) rather than the statistically-based model used in previous editions of the specifications. It is written in a form emphasizing reinforcement details (i.e., limiting bar spacing), rather than crack width~~, per se~~. Furthermore, the physical crack model has been shown to provide a more realistic estimate of crack widths for larger concrete covers compared to the previous equation (Destefano 2003).

SPECIFICATIONS

COMMENTARY

~~There appears to be little or no connection between surface crack width and corrosion. Thicker or additional cover for reinforcement will result in greater surface crack widths. These wider surface cracks are not detrimental to the corrosion protection of the reinforcement. In applying Eq. the actual clear cover should be used where the clear cover is 2.0 in. or less. Where the clear cover exceeds 2.0 in., a value of 2.0 in. should be used for calculation purposes related to Eq. . Additional cover may be regarded as added protection.~~

Equation 1 with Class 1 exposure condition is based on an assumed crack width of 0.017 IN. Previous research indicates that there appears to be little or no correlation between crack width and corrosion, however, the different classes of exposure conditions have been so defined in order to provide flexibility in the application of these provisions to meet the needs of the Owner. Class 1 exposure condition could be thought of as an upper bound in regards to crack width for appearance and corrosion. Areas that the Owner may consider for Class 2 exposure condition would include decks and substructures exposed to water. The crack width is directly proportional to the γ_e exposure factor, therefore, if the Owner desires an alternate crack width, the γ_e factor can be adjusted directly. For example a γ_e factor of 0.5 will result in an approximate crack width of 0.0085.

Where members are exposed to aggressive exposure or corrosive environments, additional protection beyond that provided by satisfying Eq. 1 may be provided by decreasing the permeability of the concrete and/or waterproofing the exposed surface.

Cracks in segmental concrete box girders may result from stresses due to handling and storing segments for precast construction and to stripping forms and supports from cast-in-place construction before attainment of the nominal f'_c.

~~The basic derivation of the crack ….~~

The β_s factor, which is a geometric relationship between the crack width at the tension face versus the crack width at the reinforcement level, has been incorporated into the basic crack control equation in order to provide uniformity of application for flexural member depths ranging from thin slabs in box culverts to deep pier caps and thick footings. The theoretical definition of β_s may be used in lieu of the approximate expression provided.

SPECIFICATIONS	COMMENTARY
5.7.3.5 MOMENT REDISTRIBUTION	
	C5.7.3.5
5.7.3.6 DEFORMATIONS	C5.7.3.6
5.7.3.6 General	C5.7.3.6
5.7.3.6.2 Deflection and Camber	C5.7.3.6.2
5.7.3.6.3 Axial Deformation	C5.7.3.6.3
5.7.4 Compression Members	C5.7.4
5.7.4.1 GENERAL	C5.7.4.1
5.7.4.2 LIMITS FOR REINFORCEMENT	C5.7.4.2
5.7.4.3 APPROXIMATE EVALUATION OF SLENDERNESS EFFECTS	C5.7.4.3
5.7.4.4 FACTORED AXIAL RESISTANCE	C5.7.4.4
5.7.4.5 BIAXIAL FLEXURE	C5.7.4.5
5.7.4.6 SPIRALS AND TIES	C5.7.4.6
5.7.4.7 HOLLOW RECTANGULAR COMPRESSION MEMBERS	C5.7.4.7
5.7.4.7.1 Wall Slenderness Ratio	C5.7.4.7.1
5.7.4.7.2 Limitations on the Use of the Rectangular Stress Block Method	C5.7.4.7.2
	C5.7.4.7.2a
5.7.4.7.2a General	
5.7.4.7.2b Refined Method for Adjusting Maximum Usable Strain Limit	C5.7.4.7.2b
5.7.4.7.2c Approximate Method for Adjusting Factored Resistance	C5.7.4.7.2c
5.7.5 Bearing	C5.7.5
5.7.6 Tension Members	C5.7.6
5.7.6.1 FACTORED TENSION RESISTANCE	C5.7.6.1
5.7.6.2 RESISTANCE TO COMBINATIONS OF TENSION AND FLEXURE	C5.7.6.2
	5.8 SHEAR AND TORSION

SPECIFICATIONS	COMMENTARY
	C5.8
5.8.1 Design Procedures	C5.8.1
5.8.1.1 FLEXURAL REGIONS	C5.8.1.1
5.8.1.2 REGIONS NEAR DISCONTINUITIES	C5.8.1.2
5.8.1.3 INTERFACE REGIONS	C5.8.1.3
5.8.1.4 SLABS AND FOOTINGS	C5.8.1.4
5.8.2 General Requirements	C5.8.2

SPECIFICATIONS
5.8.2.1 GENERAL

Add the following restriction to Equation 4:
where:

$$\frac{A_{cp}^2}{p_c} \le 2A_o b_v \quad \textit{for cellular structures}$$

Add the following as a new last paragraph:
The shear demand, V_u, shall be taken as:

- For solid sections, $V_u = \sqrt{V_u^2 + \left(\frac{0.9 p_h T_u}{2A_o}\right)^2}$ (5.8.2.1-5)
- For box sections, $V_u = V_u + \frac{T_u d_s}{2A_o}$ (5.8.2.1-6)

where:
p_h = perimeter of the centerline of the closed transverse torsion reinforcement (IN)
A_o = area enclosed by the shear flow path, including area of any holes (IN^2)
T_u = factored torsionial moment (K-IN)

The limit to Equation 4 was added to avoid over-estimating T_{cr} in the case of cellular structures. Equation 4 was derived from a solid section assuming an equivalent thin wall tube. When the actual b_v and A_{cp} is considered, torsional resistance

COMMENTARY
C5.8.2.1

can be much less. The resulting expression matches that in the *AASHTO Guide Specifications for Segmental Bridges*, 2nd Edition.

A stress limit for principal tension at the neutral axis in the web was added in 2004. This check requires shear demand, and not the resistance, to be modified for torsion. Equations 5 and 6 were added to clarify how demand is modified for torsion. Note that the V_u in Equations 5.8.3.4.2-1,2,3 for ε_x, and in Equation 5.8.2.9-1 for v_u, are no longer modified for torsion. In other words, the values used to select β, θ in Tables 5.8.3.4.2-1 and 2 have not been modified for torsion.

For solid cross-section shapes, such as a rectangle or an "I", there is the possibility of considerable redistribution of shear stresses. To make some allowance for this favorable redistribution it is safe to use a root-mean-square approach in calculating the nominal shear stress for these cross-sections, as indicated in Equation 5. The $0.9p_h$ comes from 90% of the perimeter of the spalled concrete section. This is similar to multiplying 0.9 times the lever arm in flexural calculations.

For a box girder, the shear flow due to torsion is added to the shear flow due to flexure in one exterior girder, and subtracted from the opposite exterior girder. In the controlling girder, the 2nd term in Equation 6 comes from integratng the distance from the centroid of the section, to the center of the shear flow path around the circumference of the section. The stress is converted to a force by multiplying by the girder height measured between the shear flow paths in the top and bottom slabs, which has a value approximately equal that of d_v. If the exterior girder is sloped, this distance should be divided by the sine of the girder angle from horizontal.

5.8.2.2 MODIFICATIONS FOR LOW-DENSITY CONCRETE

SPECIFICATIONS | COMMENTARY

5.8.2.3 TRANSFER AND DEVELOPMENT LENGTHS

C5.8.2.3

5.8.2.4 REGIONS REQUIRING TRANSVERSE REINFORCEMENT

C5.8.2.4

5.8.2.4 MINIMUM TRANSVERSE REINFORCEMENT

C5.8.2.4

5.8.2.6 TYPES OF TRANSVERSE REINFORCEMENT

C5.8.2.6

5.8.2.7 MAXIMUM SPACING OF TRANSVERSE REINFORCEMENT

C5.8.2.7

5.8.2.8 DESIGN AND DETAILING REQUIREMENTS

C5.8.2.8

5.8.2.9 SHEAR STRESS ON CONCRETE

Revise Eqn 5.8.2.9-1 as shown below:

$$\cancel{v = \frac{V_u - \varphi V_p}{\varphi b_v d_v}} \qquad v = \frac{\left|V_u - \varphi V_p\right|}{\varphi b_v d_v}$$

Modify paragraph 2 as follows:

In determining the web width at a particular level, one-half the diameters of ungrouted ducts or one-quarter the diameter of grouted ducts at that level shall be subtracted from the web width. It is not necessary to reduce b_v for the presence of ducts in fully grouted cast-in-place box girder frames.

5.8.3 Sectional Design Model

5.8.3.1 GENERAL

C5.8.3.1

5.8.3.2 SECTIONS NEAR SUPPORTS

C5.8.3.2

5.8.3.3 NOMINAL SHEAR RESISTANCE

C5.8.3.3

5.8.3.4 DETERMINATION OF β AND θ

C5.8.3.4

5.8.3.4.1 Simplified Procedure for Nonprestressed Sections

C5.8.3.4.1

5.8.3.4.2 General Procedure

C5.8.3.4.2

Revise Equations 5.8.3.4.2-1, 2, 3 as shown below:

SPECIFICATIONS

COMMENTARY

$$\varepsilon_x = \left[\frac{\frac{M_u}{d_v} + 0.5N_u + 0.5(V_u - V_p)\cot\theta - A_{ps} f_{po}}{2(\cdots)} \right]$$

$$\varepsilon_x = \left[\frac{\frac{|M_u|}{d_v} + 0.5N_u + 0.5(|V_u - V_p|)\cot\theta - A_{ps} f_{po}}{2(\cdots)} \right]$$

Not an amendment; approved 6/04; will appear in '05 Interims

SPECIFICATIONS

5.8.3.5 LONGITUDINAL REINFORCEMENT

Revise Equation 5.8.3.5-1 as shown below:

$$\cancel{A_{ps} f_{ps} + A_s f_y \geq \frac{M_u}{d_v \phi} + 0.5 \frac{N_u}{\phi} + \left(\frac{V_u}{\phi} - 0.5 V_s - V_p \right) cot\,\theta}$$

$$A_{ps} f_{ps} + A_s f_y \geq \frac{\left| M_u \right|}{d_v \phi} + 0.5 \frac{N_u}{\phi} + \left(\left| \frac{V_u}{\phi} - V_p \right| - 0.5 V_S \right) cot\,\theta$$

COMMENTARY

C5.8.3.5

Not an amendment; approved 6/04; will appear in '05 Interims

SPECIFICATIONS

5.8.3.6 SECTIONS SUBJECTED TO COMBINED SHEAR AND TORSION

5.8.3.6.1 Transverse Reinforcement

5.8.3.6.2 Torsional Resistance

In Article 5.8.3.6.2, modify the definition of A_t as follows: A_t = area of one leg of closed transverse torsion reinforcement <u>in solid members, or total area of transverse torsion reinforcement in the exterior girder of cellular members</u> (IN^2)

Delete paragraphs 2 and 3, including Equations 2, 3, 4.

5.8.3.6.3 Longitudinal Reinforcement

The longitudinal reinforcement <u>in solid sections</u> shall be proportioned to satisfy Equation 1.

<u>In box sections, longitudinal reinforcement for torsion, in addition to that required for flexure, shall not be less than</u>

$$\underline{A_l = \frac{T_n p_h}{2A_o f_y}} \qquad \underline{(5.8.3.6.3\text{-}2)}$$

COMMENTARY

C5.8.3.6

C5.8.3.6.1

C5.8.3.6.2

Delete paragraphs 2 and 3

C5.8.3.6.3

Not an amendment; approved 6/04; will appear in '05 Interims

SPECIFICATIONS	COMMENTARY
5.8.4 Interface Shear Transfer – Shear Friction	C5.8.4
5.8.4.1 GENERAL	
	C5.8.4
5.8.4.2 COHESION AND FRICTION	C5.8.4.2
5.8.5 Direct Shear Resistance of Dry Joints	C5.8.5
5.9 PRESTRESSING AND PARTIAL PRESTRESSING	C5.9
5.9.1 General Design Considerations	C5.9.1
5.9.1.1 GENERAL	C5.9.1.1
5.9.1.2 SPECIFIED CONCRETE STRENGTHS	C5.9.1.2
5.9.1.3 BUCKLING	C5.9.1.3
5.9.1.4 SECTION PROPERTIES	C5.9.1.4
5.9.1.5 CRACK CONTROL	C5.9.1.5
5.9.1.6 TENDONS WITH ANGLE POINTS OR CURVES	C5.9.1.6
5.9.2 Stresses Due to Imposed Deformation	C5.9.2

SPECIFICATIONS

5.9.3 Stress Limitations for Prestressing Tendons

The following entry in Table 5.9.3-1 shall be changed from:

Prior to Seating	$0.90f_{py}$	$0.90f_{py}$	$0.90f_{py}$

to

Maximum Jacking Stress	$0.90f_{py}$	<u>$0.75f_{pu}$ (see note)</u>	$0.90f_{py}$

The following shall be added below Table 5.9.3-1:

<u>Note: For longer frame structures, tensioning to $0.90f_{py}$ for short periods of time prior to seating may be permitted to offset seating and friction losses provided the stress at the anchorage does not exceed the above value (low relaxation strand, only).</u>

5.9.4.1 FOR TEMPORARY STRESSES BEFORE LOSSES – FULLY PRESTRESSED COMPONENTS

5.9.4.1.1 Compression Stresses

5.9.4.1.2 Tension Stresses

COMMENTARY

C5.9.3

5.9.4 Stress Limits for Concrete

C5.9.4

C5.9.4.1

C5.9.4.1.1

C5.9.4.1.2

SPECIFICATIONS

5.9.4.2 FOR STRESSES AT SERVICE LIMIT STATE AFTER LOSSES – FULLY PRESTRESSED COMPONENTS

5.9.4.2.1 Compression Stresses

5.9.4.2.2 Tension Stresses

The following shall replace entries in Table 5.9.4.2.2-1 for “other than segmentally constructed bridges”.

COMMENTARY

C5.9.4.2.1

C5.9.4.2.2

SPECIFICATIONS COMMENTARY

Table 5.9.4.2.2-1 Tensile Stress Limits in Prestressed concrete at Service Limit State After Losses, Fully Prestressed Components

Bridge Type	Location	Stress Limit
Other Than Segmentally Constructed Bridges	Tension in the Precompressed Tensile Zone Bridges, Assuming Uncracked Sections	
	• For components with bonded prestressing tendons or reinforcement, subjected to permanent loads, only.	No tension
	• For components with bonded prestressing tendons or reinforcement that are subjected to not worse than moderate corrosion conditions, and are located in Caltrans Environmental Areas I or II.	$0.19\sqrt{f'c}$ (KSI) $0.50\sqrt{f'c}$ (MPa)
	• For components with bonded prestressing tendons or reinforcement that are subjected to severe corrosive conditions, and are located in Caltrans Environmental Area III.	$0.0948\sqrt{f'c}$ (KSI) $0.25\sqrt{f'c}$ (MPa)
	• For components with unbonded prestressing tendons.	No tension

SPECIFICATIONS

COMMENTARY

5.9.5 Loss of Prestress

C5.9.5

5.9.5.1 TOTAL LOSS OF PRESTRESS

C5.9.5.1

Values of prestress losses specified herin shall be applicable for specified concrete strengths up to 15.0 KSI. In lieu of more detailed analysis, prestress losses in members prestressed in a single stage, relative to the stress immediately before transfer may be taken as:

- In pretensioned members $\Delta f_{PT} = \Delta f_{pES} + \Delta f_{pLT}$ (5.9.5.1-1)
- In post-tensioned members $\Delta f_{pT} = \Delta f_{pF} + \Delta f_{pA} + \Delta f_{pES} + \Delta f_{pLT}$ (5.9.5.1-2)

5.9.5.2 INSTANANEOUS LOSSES

C5.9.5.2

5.9.5.2.1 Anchorage Set

C5.9.5.2.1

5.9.5.2.2 Friction

C5.9.5.2.2

5.9.5.2.2a Pretensioned Construction

C5.9.5.2.2a

5.9.5.2.2b Posttensioned Construction

C5.9.5.2.2b

5.9.5.2.3 Elastic Shortening

C5.9.5.2.3

5.9.5.2.3a Pretensioned Members

C5.9.5.2.3a

5.9.5.2.3b Posttensioned Members

C5.9.5.2.3

SPECIFICATIONS

5.9.5.3 APPROXIMATE ~~LUMP SUM~~ ESTIMATE OF TIME-DEPENDENT LOSSES

Add the following to the end of Article 5.9.5.3:

For ~~cast-in-place~~ post-tensioned members, the approximate estimate of time-dependent losses may be taken as the lump sum value of 25 ksi.

COMMENTARY

C5.9.5.3

Add the following to the end of C5.9.5.3:

The expression for I-girders was replaced to better match field data using high strength concrete as reported in NCHRP 496. The new approximation is the lump sum estimate of effects due to shrinkage, creep, and relaxation (abbreviated from *'05 Interims*).

The expressions for estimating time-dependent losses in Table 5.9.5.3-1 were developed for pretentioned ~~precast~~ members and should not be used for ~~cast-in-place~~ post-tensioned structures. Preliminary research at UCSD indicates that the time-dependent losses for cast-in-place post-tensioned structures are between 25 ksi and 30 ksi. Until the research is completed, and, in lieu of a more detailed analysis, a lump sum value for losses in post-tensioned members is provided.

Table 5.9.5.3-1 Time-Dependent Losses for Pretensioned Members in ksi.

Type of Beam Section	Level	For Wires and Strands with f_{pu} = 235, 250 or 270 ksi	For Bars with f_{pu} = 145 or 160 ksi
Rectangular Beams, Solid Slab	Upper Bound Average	29.0 + 4.0 *PPR* 26.0 + 4.0 *PPR*	19.0 + 6.0 *PPR*
Box Girder	Upper Bound Average	21.0 + 4.0 *PPR* 19.0 + 4.0 *PPR*	15.0
I-Girder	Average	~~$33.0\left[1.0-0.15\frac{f'_c-6.0}{6.0}\right]+6.0\,PPR$~~ $10.0\frac{f_{pi}A_{ps}}{A_g}\gamma_h\gamma_{st}+12.0\gamma_{st}\gamma_h+\Delta f_{pR}$ where γ_h = 1.7-0.01H, γ_{st}= 5/(1+f_{ci}), Δf_{pR} = 2.5	19.0 + 6.0 *PPR*
Single T, Double T, Hollow Core and Voided Slab	Upper Bound Average	$39.0\left[1.0-0.15\frac{f'_c-6.0}{6.0}\right]+6.0\,PPR$ $33.0\left[1.0-0.15\frac{f'_c-6.0}{6.0}\right]+6.0\,PPR$	$31.0\left[1.0-0.15\frac{f'_c-6.0}{6.0}\right]+6.0\,PPR$

SPECIFICATIONS

5.9.5.4 REFINED ESTIMATES OF TIME-DEPENDENT LOSSES Replaced in 2004. Outline shown below.

5.9.5.4.1 General

$$\Delta f_{pLT} = \left(\Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR1}\right)_{id} + \left(\Delta f_{pSD} + \Delta f_{pCD} + \Delta f_{pR2} - \Delta f_{pSS}\right)_{df} \quad (5.9.5.4.1\text{-}1)$$

5.9.5.4.2 ~~Shrinkage~~ Time of Transfer to Time of Deck Placement

5.9.5.4.3 ~~Creep~~ Time of Deck Placement to Final Time

5.9.5.4.4 ~~Relaxation~~ Precast Pretensioned Girders without Composite Topping

5.9.5.4.5 Post-tensioned Non-segmental Girders

Long-term prestress losses for post-tensioned members after tendons have been grouted may be calculated using the provisions of Articles 5.9.5.4.1 through 5.9.5.4.1-4. In Eq. 5.9.5.4.1-1, the value of the term $(\Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR1})_{id}$ shall be taken as zero.

5.9.5.5 LOSSES FOR DEFLECTION CALCULATIONS

COMMENTARY

C5.9.5.4

C5.9.5.5

SPECIFICATIONS	COMMENTARY
5.10 DETAILS OF REINFORCEMENT	C5.10
5.10.1 Concrete Cover	C5.10.1
5.10.2 Hooks and Bends	C5.10.2
5.10.2.1 STANDARD HOOKS	C5.10.2.1
5.10.2.2 SEISMIC HOOKS	C5.10.2.2
5.10.2.3 MINIMUM BEND DIAMETERS	C5.10.2.3
5.10.3 Spacing of Reinforcement	C5.10.3
5.10.3.1 MINIMUM SPACING OF REINFORCING BARS	C5.10.3.1
	C5.10.3.1.1
5.10.3.1.1 Cast-in-Place Concrete	
5.10.3.1.2 Precast Concrete	C5.10.3.1.2
5.10.3.1.3 Multilayers	C5.10.3.1.3
5.10.3.1.4 Splices	C5.10.3.1.4
5.10.3.1.5 Bundled Bars	C5.10.3.1.5
5.10.3.2 MAXIMUM SPACING OF REINFORCING BARS	C5.10.3.2
5.10.3.3 MINIMUM SPACING OF PRESTRESSING TENDONS AND DUCTS	C5.10.3.3
5.10.3.3.1 Pretensioning Strand	C5.10.3.3.1
5.10.3.3.2 Posttensioning Ducts Not Curved in the Horizontal Plane	C5.10.3.3.2
5.10.3.3.3 Curved Posttensioning Ducts	C5.10.3.3.3
5.10.3.4 MAXIMUM SPACING OF PRESTRESSING TENDONS AND DUCTS IN SLABS	C5.10.3.4
5.10.3.5 COUPLERS IN POSTTENSIONING TENDONS	C5.10.3.5
5.10.4 Tendon Confinement	C5.10.4

SPECIFICATIONS	COMMENTARY
5.10.4.1 GENERAL	C5.10.4.1
5.10.4.2 WOBBLE EFFECT IN SLABS	C5.10.4.2
5.10.4.3 EFFECTS OF CURVED TENDONS	C5.10.4.3
5.10.4.3.1 In-Plane Force Effects	C5.10.4.3.1
5.10.4.3.2 Out-of-Plane Force Effects	C5.10.4.3.2
5.10.5 External Tendon Supports	C5.10.5
5.10.6 Transverse Reinforcement for Compression members	C5.10.6
5.10.6.1 GENERAL	C5.10.6.1
5.10.6.2 SPIRALS	C5.10.6.2
5.10.6.3 TIES	C5.10.6.3
5.10.7 Transverse Reinforcement for Flexural Members	C5.10.7
5.10.8 Shrinkage and Temperature Reinforcement	C5.10.8
5.10.8.1 GENERAL	C5.10.8.1
5.10.8.2 COMPONENTS LESS THAN 1200 mm THICK	C5.10.8.2
5.10.8.3 MASS CONCRETE	C5.10.8.3

SPECIFICATIONS	COMMENTARY
5.10.9 Posttensioned Anchorage Zones	C5.10.9
5.10.9.1 GENERAL	C5.10.9.1
5.10.9.2 GENERAL ZONE AND LOCAL ZONE	C5.10.9.2
5.10.9.2.1 General	C5.10.9.2.1
5.10.9.2.2 General Zone	C5.10.9.2.2
5.10.9.2.3 Local Zone	C5.10.9.2.3
5.10.9.2.4 Responsibilities	C5.10.9.2.4
5.10.9.3 DESIGN OF THE GENERAL ZONE	C5.10.9.3
5.10.9.3.1 Design Methods	C5.10.9.3.1
5.10.9.3.2 Design Principles	C5.10.9.3.2
5.10.9.3.3 Special Anchorage Devices	C5.10.9.3.3
5.10.9.3.4 Intermediate Anchorages	C5.10.9.3.4
5.10.9.3.4a General	C5.10.9.3.4a
5.10.9.3.4b Tie Backs	C5.10.9.3.4b
5.10.9.3.4c Blister and Rib Reinforcement	C5.10.9.3.4c
5.10.9.3.5 Diaphragms	C5.10.9.3.5
5.10.9.3.6 Multiple Slab Anchorages	C5.10.9.3.6
5.10.9.3.7 Deviation Saddles	C5.10.9.3.7
5.10.9.4 APPLICATION OF THE STRUT-AND-TIE MODEL TO THE DESIGN OF GENERAL ZONE	C5.10.9.4
5.10.9.4.1 General	C5.10.9.4.1
5.10.9.4.2 Nodes	C5.10.9.4.2
5.10.9.4.3 Struts	C5.10.9.4.3

SPECIFICATIONS	COMMENTARY
5.10.9.4.4 Ties	C5.10.9.4.4
5.10.9.5 ELASTIC STRESS ANALYSIS	C5.10.9.5
5.10.9.6 APPROXIMATE STRESS ANALYSES AND DESIGN	C5.10.9.6
5.10.9.6.1 Limitations of Application	C5.10.9.6.1
5.10.9.6.2 Compressive Stresses	C5.10.9.6.2
5.10.9.6.3 Bursting Forces	C5.10.9.6.3
5.10.9.6.4 Edge Tension Forces	C5.10.9.6.4
5.10.9.7 DESIGN OF LOCAL ZONES	C5.10.9.7
5.10.9.7.1 Dimensions of Local Zone	C5.10.9.7.1
5.10.9.7.2 Bearing Resistance	C5.10.9.7.2
5.10.9.7.3 Special Anchorage Devices	C5.10.9.7.3
5.10.10 Pretensioned Anchorage Zones	C5.10.10
5.10.10.1 FACTORED BURSTING RESISTANCE	C5.10.10.1
5.10.10.2 CONFINEMENT REINFORCEMENT	C5.10.10.2
5.10.11 Provisions for Seismic Design	C5.10.11
5.10.11.1 GENERAL	C5.10.11.1
5.10.11.2 SEISMIC ZONE 1	C5.10.11.2
5.10.11.3 SEISMIC ZONE 2	C5.10.11.3
5.10.11.4 SEISMIC ZONES 3 AND 4	C5.10.11.4
5.10.11.4.1 Column Requirements	C5.10.11.4.1
5.10.11.4.1a Longitudinal Reinforcement	C5.10.11.4.1a
5.10.11.4.1b Flexural Resistance	C5.10.11.4.1b
5.10.11.4.1c Column Shear and Transverse Reinforcement	C5.10.11.4.1c

SPECIFICATIONS

5.10.11.4.1d Transverse Reinforcement for Confinement at Plastic Hinges

Modify paragraph 2 as follows:

For a circular column, the volumetric ratio of spiral reinforcement, ρ_s, shall satisfy ~~either that required in Article 5.7.4.6 or~~:

$$\cancel{\rho_s \geq 0.12 \frac{f'_c}{f_y}} \qquad \cancel{(5.10.11.4.1d\text{-}1)}$$

$$\rho_s \geq 0.45\left(\frac{A_g}{A_c}-1\right)\frac{f'_c}{f_y}\left(0.5+\frac{1.25P_e}{f'_c A_g}\right) \qquad (5.10.11.4.1d\text{-}1a)$$

for columns less than or equal to 3 feet in diameter or least dimension, or

$$\rho_s \geq 0.12\frac{f'_c}{f_y}\left(0.5+\frac{1.25P_e}{f'_c A_g}\right) \qquad (5.10.11.4.1d\text{-}1b)$$

for columns larger than 3 feet in diameter or least dimension. However ρ_s shall not be less than that required by Eq. 5.7.4.6-1. If the cover over the core in a plastic hinge zone exceeds two inches at any point, the value of A_g used to determine ρ_s for the plastic hinge zone shall be limited to the area of a reduced section having not more than two inches of cover. The reduced section used to calculate A_g shall be adequate for all applied loads associated with the plastic hinge.

5.10.11.4.1e Spacing of Transverse Reinforcement for Confinement

5.10.11.4.1f Splices

5.10.11.4.2 Requirements for Wall-Type Piers

5.10.11.4.3 Column Connections

5.10.11.4.4 Construction Joints in Piers and Columns

COMMENTARY

C5.10.11.4.1d

C5.10.11.4.1e

C5.10.11.4.1f

C5.10.11.4.2

C5.10.11.4.3

C5.10.11.4.4

SPECIFICATIONS	COMMENTARY
5.10.12 Reinforcement for Hollow Rectangular Compression Members	C5.10.12
5.10.12.1 GENERAL	C5.10.12.1
5.10.12.2 SPACING OF REINFORCEMENT	C5.10.12.2
5.10.12.3 TIES	C5.10.12.3
5.10.12.4 SPLICES	C5.10.12.4
5.10.12.5	C5.10.12.5
5.11 DEVELOPMENT AND SPLICES OF REINFORCEMENT	C5.11
5.11.1 General	C5.11.1
5.11.1.1 BASIC REQUIREMENTS	C5.11.1.1
5.11.1.2 FLEXURAL REINFORCEMENT	C5.11.1.2
5.11.1.2.1 General	C5.11.1.2.1
5.11.1.2.2 Positive Moment Reinforcement	C5.11.1.2.2
5.11.1.2.3 Negative Moment Reinforcement	C5.11.1.2.3
5.11.1.2.4 Moment Resisting Joints	C5.11.1.2.4
5.11.2 Development of Reinforcement	C5.11.2
5.11.2.1 DEFORMED BARS AND DEFORMED WIRE IN TENSION	C5.11.2.1
5.11.2.1.1 Tension Development Length	C5.11.2.1.1
5.11.2.1.2 Modification Factors That Increase	C5.11.2.1.2
5.11.2.1.3 Modification Factors That Decrease	C5.11.2.1.3
5.11.2.2 DEFORMED BARS IN COMPRESSION	C5.11.2.2
5.11.2.2.1 Compressive Development Length	C5.11.2.2.1
5.11.2.2.2 Modification Factors	C5.11.2.2.2
5.11.2.3 BUNDLED BARS	C5.11.2.3
5.11.2.4 STANDARD HOOKS IN TENSION	C5.11.2.4
5.11.2.4.1 Basic Hook Development Length	C5.11.2.4.1
5.11.2.4.2 Modification Factors	C5.11.2.4.

SPECIFICATIONS	COMMENTARY
5.11.2.4.3 Hooked-Bar Tie Requirements	C5.11.2.4.3
5.11.2.5 WELDED WIRE FABRIC	C5.11.2.5
5.11.2.5.1 Deformed Wire Fabric	C5.11.2.5.1
5.11.2.5.2 Plain Wire Fabric	C5.11.2.5.2
5.11.2.6 SHEAR REINFORCEMENT	C5.11.2.6
5.11.2.6.1 General	C5.11.2.6.1
5.11.2.6.2 Anchorage of Deformed Reinforcement	C5.11.2.6.2
5.11.2.6.3 Anchorage of Wire Fabric Reinforcement	C5.11.2.6.3
5.11.2.6.4 Closed Stirrups	C5.11.2.6.4
5.11.3 Development by Mechanical Anchorages	C5.11.3

SPECIFICATIONS

5.11.4 Development of Prestressing Strand

5.11.4.1 GENERAL

5.11.4.2 BONDED STRAND

5.11.4.3 PARTIALLY DEBONDED STRANDS

The number of partially debonded strands should not exceed ~~25~~ 33 percent of the total number of strands.

The number of debonded strands in any horizontal row shall not exceed ~~40~~ 50 percent of the strands in that row.

The length of debonding of any strand shall be such that all limit states are satisfied with consideration of the total developed resistance at any section being investigated. Not more than 40 percent of the debonded strands, or four strands, whichever is greater, shall have the debonding terminated at any section.

5.11.5 Splices of Bar Reinforcement

5.11.5.1 DETAILING

5.11.5.2 GENERAL REQUIREMENTS

5.11.5.2.1 Lap Splices

5.11.5.2.2 Mechanical Connections

5.11.5.2.3 Welded Splices

5.11.5.3 SPLICES OF REINFORCMENT IN TENSION

5.11.5.3.1 Lap Splices in Tension

5.11.5.3.2 Mechanical Connections or Welded Splices in Tension

5.11.5.4 SPLICES IN TENSION TIE MEMBERS

5.11.5.5 SPLICES OF BARS IN COMPRESSION

5.11.5.5.1 Lap Splices in Compression

5.11.5.5.2 Mechanical Connections or Welded Splices in Compression

5.11.5.5.3 End-Bearing Splices

5.11.6 Splices of Welded Wire Fabric

COMMENTARY

C5.11.4

C5.11.4.1

C5.11.4.2

C5.11.4.3

Tests completed by the Florida Department of Transportation (*Shahawy, Robinson, and Batchelor 1993*), (*Shahawy and Batchelor 1991*) indicate that the anchored strength of the strands is one of the primary contributors to the shear resistance of prestressed concrete beams in their end zones. The recommended limit of 25 percent of debonded strands is derived from those tests. Shear capacity was found to be inadequate with full-scale girders where 40 percent of strands were debonded.

C5.11.5

C5.11.5.1

C5.11.5.2

C5.11.5.2.1

C5.11.5.2.2

C5.11.5.2.3

C5.11.5.3

C5.11.5.3.1

C5.11.5.3.2

C5.11.5.4

C5.11.5.5

C5.11.5.5.1

C5.11.5.5.2

C5.11.5.5.3

C5.11.6

5.11.6.1 SPLICES OF WELDED DEFORMED WIRE FABRIC IN TENSION

SPECIFICATIONS	COMMENTARY
	C5.11.6.1
5.11.6.2 SPLICES OF WELDED SMOOTH WIRE FABRIC IN TENSION	C5.11.6.2
5.12 DURABILITY	C5.12
5.12.1 General	C5.12.1
5.12.2 Alkali-Silica Reactive Aggregates	C5.12.2
5.12.3 Concrete Cover	C5.12.3
5.12.4 Protective Coatings	C5.12.4
5.12.5 Protection for Prestressing Tendons	C5.12.5
5.13 SPECIFIC MEMBERS	C5.13
5.13.1 Deck Slabs	C5.13.1
5.13.2 Diaphragms, Deep Beams, Brackets, Corbels and Beam Ledges	C5.13.2
5.13.2.1 GENERAL	C5.13.2.1
5.13.2.2 DIAPHRAGMS	C5.13.2.2
5.13.2.3 DETAILING REQUIREMENTS FOR DEEP BEAMS	C5.13.2.3
5.13.2.4 BRACKETS AND CORBELS	C5.13.2.4
5.13.2.4.1 General	C5.13.2.4.1
5.13.2.4.2 Alternative to Strut-and-Tie Model	C5.13.2.4.2
5.13.2.5 BEAM LEDGES	C5.13.2.5
5.13.2.5.1 General	C5.13.2.5.1
5.13.2.5.2 Design for Shear	C5.13.2.5.2
5.13.2.5.3 Design for Flexure and Horizontal Force	C5.13.2.5.3
5.13.2.5.4 Design for Punching Shear	C5.13.2.5.4
5.13.2.5.5 Design of Hanger Reinforcement	C5.13.2.5.5
5.13.2.5.6 Design for Bearing	C5.13.2.5.6

SPECIFICATIONS	COMMENTARY
5.13.3 Footings	C5.13.3
5.13.3.1 GENERAL	C5.13.3.1
5.13.3.2 LOADS AND REACTIONS	C5.13.3.2
5.13.3.3 RESISTANCE FACTORS	C5.13.3.3
5.13.3.4 MOMENT IN FOOTINGS	C5.13.3.4
5.13.3.5 DISTRIBUTION OF MOMENT REINFORCEMENT	C5.13.3.5
5.13.3.6 SHEAR IN SLABS AND FOOTINGS	C5.13.3.6
5.13.3.6.1 Critical Sections for Shear	C5.13.3.6.1
5.13.3.6.2 One-Way Action	C5.13.3.6.2
5.13.3.6.3 Two-Way Action	C5.13.3.6.3
5.13.3.7 DEVELOPMENT OF REINFORCEMENT	C5.13.3.7
5.13.3.8 TRANSFER OF FORCE AT BASE OF COLUMN	C5.13.3.8
5.13.4 Concrete Piles	C5.13.4
5.13.4.1 GENERAL	C5.13.4.1
5.13.4.2 SPLICES	C5.13.4.2
5.13.4.3 PRECAST REINFORCED PILES	C5.13.4.3
5.13.4.3.1 Pile Dimensions	C5.13.4.3.1
5.13.4.3.2 Reinforcing Steel	C5.13.4.3.2
5.13.4.4 PRECAST PRESTRESSED PILES	C5.13.4.4
5.13.4.4.1 Pile Dimensions	C5.13.4.4.1
5.13.4.4.2 Concrete Quality	C5.13.4.4.2
5.13.4.4.3 Reinforcement	C5.13.4.4.3

SPECIFICATIONS

5.13.4.5 CAST-IN-PLACE PILES

5.13.4.5.1 Pile Dimensions

5.13.4.5.2 Reinforcing Steel

Insert the following as paragraph 3 in Article 5.13.4.5.2 (on AASHTO '05 ballot):

For cast-in-place concrete piling, clear distance between parallel longitudinal and parallel transverse reinforcing bars shall not be less than 5 times the maximum aggregate size or 5 inches, except as noted in Article 5.13.4.6 for seismic requirements.

COMMENTARY

C5.13.4.5

C5.13.4.5.1

C5.13.4.5.2

SPECIFICATIONS

5.13.4.6 SEISMIC REQUIREMENTS

5.13.4.6.1 Zone 1

5.13.4.6.2 Zone 2

5.13.4.6.2a General

5.13.4.6.2b Cast-in-Place Piles

Revise Article 5.13.4.6.2b as follows (on ’05 AASHTO ballot):

"For cast-in-place piles, longitudinal steel shall be provided….For piles less than 24-IN in diameter, ~~S~~spiral reinforcement or equivalent ties of not less than No. 3 bars shall be provided at a pitch not exceeding 9.0 IN, except that the pitch shall not exceed ~~3~~ 4.0 IN within a length below the pile cap reinforcement of not less than 2.0 FT or 1.5 pile diameters , whichever is greater ~~below the pile cap reinforcement~~. See Article 5.10.11.3.

5.13.4.6.2c Precast Reinforced Piles

5.13.4.6.2d Precast Prestressed Piles

5.13.4.6.3 Zones 3 and 4

5.13.4.6.3a General

COMMENTARY

C5.13.4.6

C5.13.4.6.1

C5.13.4.6.2

C5.13.4.6.2a

C5.13.4.6.2b

Add the following as C5.13.4.6.2b

Cast-in-place concrete pilings may only have been vibrated directly beneath the pile cap, or in the uppermost sections. Where concrete is not vibrated, nondestructive tests in the State of California have shown that voids and rock pockets form when adhering to maximum confinement steel spacing limitations from some seismic recommendations. Concrete does not readily flow through the resulting clear distances between bar reinforcing, weakening the concrete section, and compromising the bending resistance to lateral seismic loads. Instead of reduced bar spacing, bar diameters should be increased which results in larger openings between the parallel longitudinal and transverse reinforcing steel.

C5.13.4.6.2

C5.13.4.6.2d

C5.13.4.6.3

C5.13.4.6.3a

SPECIFICATIONS

COMMENTARY

5.13.4.6.3b Confinement Length

C5.13.4.6.3b

5.13.4.6.3c Volumetric Ratio for Confinement

C5.13.4.6.3c

5.13.4.6.3d Cast-in-Place Piles

Revise Article 5.13.4.6.3d as follows(on ’05 AASHTO ballot):

"For cast-in-place piles, longitudinal steel shall be provided….For piles less than 24-IN in diameter, ~~S~~spiral reinforcement or equivalent ties of not less than No. 3 bars shall be provided at a pitch not exceeding 9.0-IN ~~pitch~~, except that the pitch shall not exceed 4.0 IN ~~for the top~~ within a length below the pile cap reinforcement of not less than 4.0 FT or two pile diameters, whichever is greater, ~~where the pitch shall be 3 4.0 IN~~ and where the volumetric ratio and splice details shall conform to Articles 5.10.11.4.1d and e.”

C5.13.4.6.3d

Add the following as C5.13.4.6.3d

Cast-in-place concrete pilings may only have been vibrated directly beneath the pile cap, or in the uppermost sections. Where concrete is not vibrated, nondestructive tests in the State of California have shown that voids and rock pockets form when adhering to maximum confinement steel spacing limitations from some seismic recommendations. Concrete does not readily flow through the resulting clear distances between bar reinforcing, weakening the concrete section, and compromising the bending resistance to lateral seismic loads. Instead of reduced bar spacing, bar diameters should be increased which results in larger openings between the parallel longitudinal and transverse reinforcing steel.

5.14 PROVISIONS FOR STRUCTURE TYPES

C5.14

5.14.1 Beams and Girders

C5.14.1

5.14.1.2 Precast Beams

C5.14.1.2

5.14.1.2.1 Preservice Conditions

C5.14.1.2.1

5.14.1.2.7b Reinforcement

Delete the first sentence.

In the second sentence, replace “Such reinforcement” with “Reinforcement”.

C5.14.1.2.7b

Deleted.